CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

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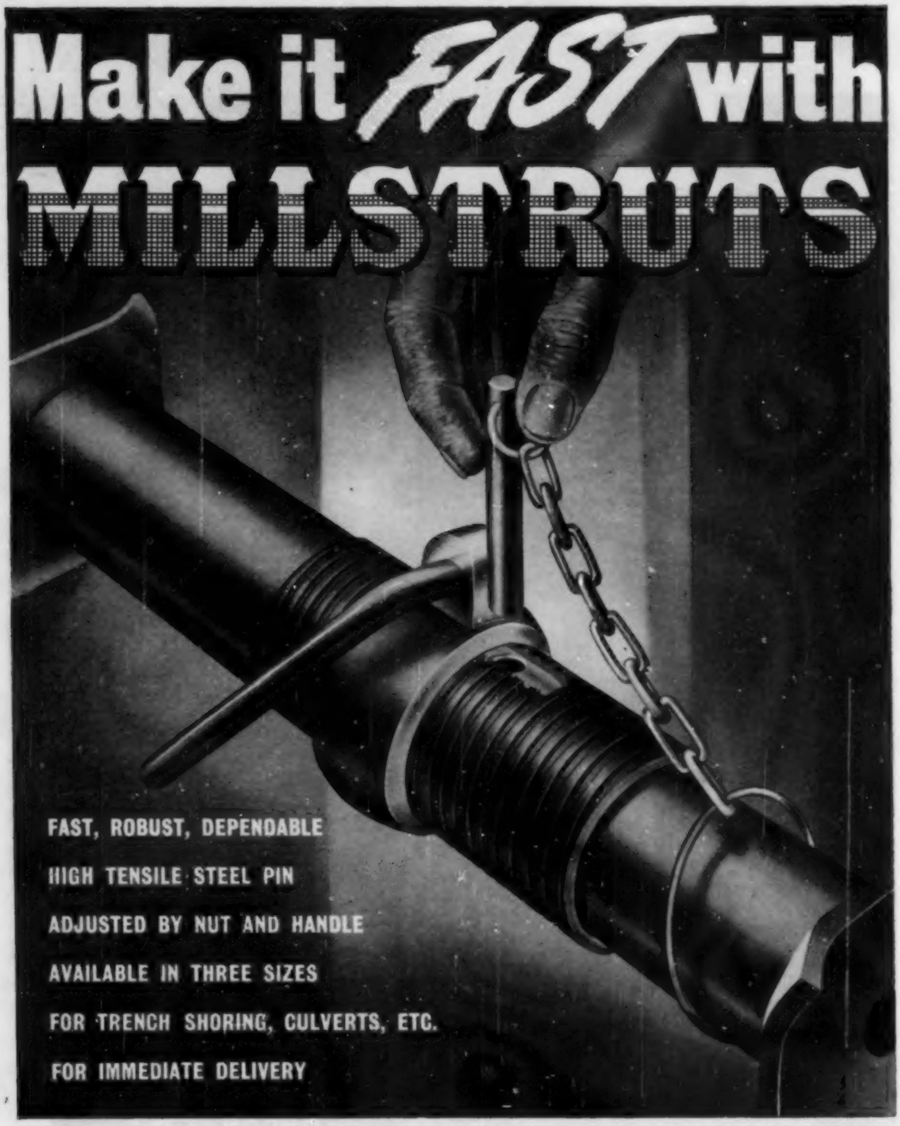
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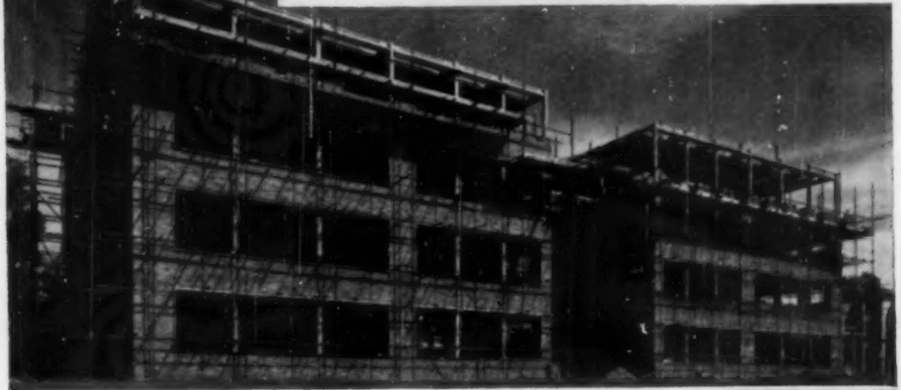

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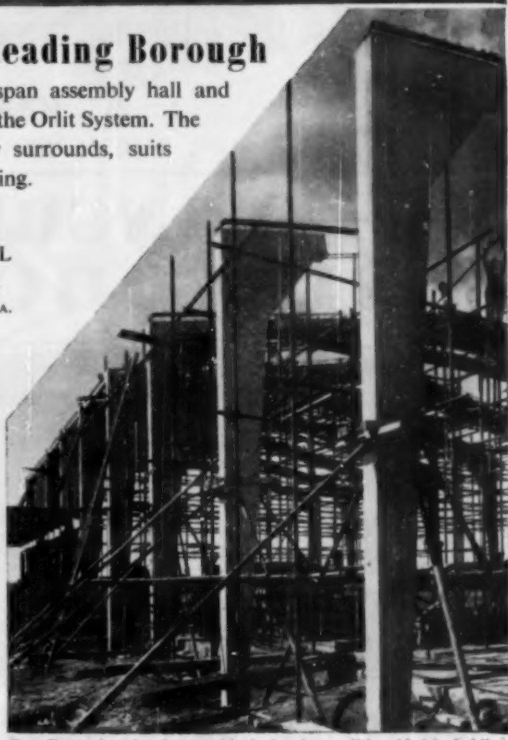

This four storey school with its 48′ span assembly hall and 42′ span gymnasium is constructed on the Orlit System. The Orlit framework, with Orlit window surrounds, suits perfectly the "traditional" brick cladding.

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Top: Front elevation 4-storey block showing traditional brick cladding and Orlit structural framework (upper storey); window surrounds by Orlit.
Below: Precast columns to 48′ span Assembly Hall.

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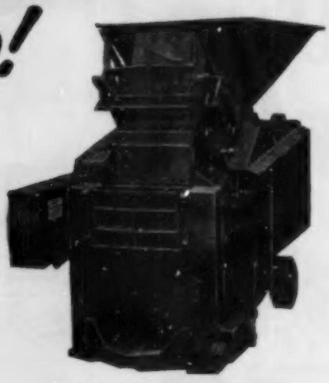

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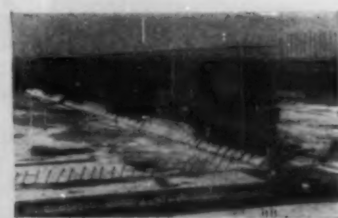

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CONCRETE
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CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

Volume XLIX, No. 11. LONDON, NOVEMBER, 1954.

EDITORIAL NOTES

Design, Decoration, and Utility.

THERE is a tendency nowadays in artistic matters for the word design to mean line only, with the consequent elimination of the artist and of decoration, and this is affecting architecture and everyday things. An example is to be seen in new designs for lamp posts approved by the Council of Industrial Design. In every case the so-called improvement consists of the elimination of decoration and the use of new shapes. The lower parts of four of the posts commented on by the Council of Industrial Design are shown on page 332. The first photograph shows the base of a decorated post and the next the base of a plain one; the Council of Industrial Design invites us to note the simplification of the design, and points out that " there is no structural need for the elaborate design shown on the left ". Comparing the post shown in the fourth photograph with that shown on its left, we are asked to " note the neat way in which the control gear has been housed in the base of the column on the right ". It seems that the Council is much over-stating its case for plainness. No one will claim that there is any structural need for the enlarged and fluted bases of the posts now criticised, although experience suggests that a little extra cover to the steel may be an advantage; but few would describe these decorations as elaborate. Nor would everyone agree that the black patch at eye level is a " neat " cover for the control gear compared with the unobtrusive door now out of favour. The confusion of thought amongst the eliminators of decoration is shown by a statement in the journal published by the Council, in which it is claimed that lamp posts are now available " devoid of unnecessary ornament ". As no ornament can ever be really necessary, it appears that the Council is also of the opinion that neither is any ornament desirable. This is indeed a confession of failure, suggesting that we have no artists who are capable of designing suitable decoration and that consequently all lamp standards must have the appearance of drain pipes on end. The Council appears to have an unusual idea of the meaning of the much-abused word " democratic ". It has itself selected a committee of seven to advise manufacturers of lamp posts and to approve designs, and in the near future posts approved by this committee will alone be eligible for a contribution from the Government for the lighting of trunk-roads; this procedure is claimed to be a " typically democratic instrument for solving a national problem, steering clear of sanctions, controls, centralisation, and standardisation ". But what is

the withholding of a financial contribution but a sanction against a local authority that uses a lamp post that is not approved by this committee? What is this restriction of choice if it is not centralisation and control and standardisation? The Council is very concerned at the extra cost that might be incurred if an architect of a local authority should design a lamp post for his own district, but already the Council has approved more than three hundred designs without apparently being unduly worried about the cost of so many different shapes.

With a material such as concrete it cannot be reasonably claimed that decoration is omitted on the ground of economy. Most lamp posts are now made by a spinning process in steel moulds, and the extra cost of a post with some decoration would be negligible in view of the large number of posts that can be made with one mould. It appears indeed that design has now come to mean utility, plainness, and sameness. This is a pity, for men do not live by bread alone and it is wrong to deny us a little pleasing decoration in order to gratify the few who believe that design means only line. The design of a lamp post is perhaps a small thing, but this standardisation, this elimination of decoration because a small and specially-selected committee thinks it is not good for us, is one more step towards the standardised life and the denial of individuality. Standards are seldom standards of excellence; they may be standards of mediocrity or utility, or they may even represent what is easiest and cheapest to make with the plant and facilities available to the most poorly-equipped member of a trade association. It would be easy to appoint a committee of artists and architects that would have produced entirely different designs, and who would not have been afraid to use decoration to provide some relief to these plain posts. The Council may claim that it is not against decoration, but these new designs suggest that at any rate it is not in favour of it. Some of the earlier concrete lamp posts, and particularly their brackets, were undoubtedly over-decorative, wasteful of material, and expensive to make, but this is not a sound reason for denying some decoration at little extra cost.

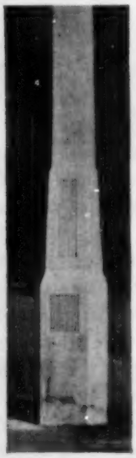

Fig. 1.

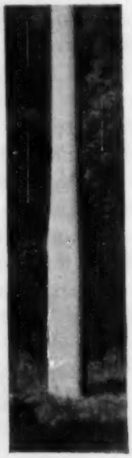

Fig. 2.

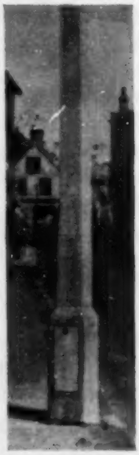

Fig. 3.

Fig. 4.

Doubly-reinforced Beams.

A Quick Method of Design.

By J. S. SAVONA, A.M.I.Struct.E.

In *Table* I and the graph in *Fig.* 2 is given a quick method of designing or checking the stresses in doubly-reinforced beams.

Table I.—The symbols used in the calculations conform with standard British practice. *Fig.* 1 shows a section of a beam reinforced with an area of

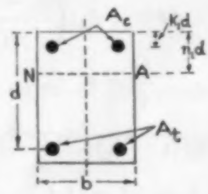

Fig. 1.

steel A_c sq. in. at the top and A_t sq. in. at the bottom where $A_c = p'.bd$, $A_t = p.bd$, and $\frac{A_c}{A_t} = \frac{p'}{p} = \phi$. Equating the total compression to the total tension,

$$\frac{c}{2} \times bn_1d + c\left(\frac{n_1 - k_1}{n_1}\right)(m-1)\phi p.bd = mc\left(\frac{1 - n_1}{n_1}\right)p.bd \quad . \quad (1)$$

Dividing by $c.bd$ and rearranging, this becomes

$$n_1^2 + 2p[\phi(m-1) + m]n_1 - 2p[\phi k_1(m-1) + m] = 0 \quad . \quad . \quad (2)$$

Solving for n_1 in (2),

$$n_1 = \frac{-2p[\phi(m-1)+m] \pm \sqrt{[2p\{\phi(m-1)+m\}]^2 + 4 \times 2p[\phi k_1(m-1)+m]}}{2} \quad . \quad (2a)$$

If

$$2p[\phi(m-1) + m] = A \quad . \quad . \quad . \quad . \quad . \quad (3)$$

$$8p[\phi k_1(m-1) + m] = B \quad . \quad . \quad . \quad . \quad . \quad (4)$$

then

$$n_1 = \frac{-A \pm \sqrt{A^2 + B}}{2} \quad . \quad . \quad . \quad . \quad . \quad (2b)$$

Rearranging (3) and (4),

$$\frac{A}{p} = 2[\phi(m-1) + m] \quad . \quad . \quad . \quad . \quad . \quad (3a)$$

$$\frac{B}{p} = 8[\phi k_1(m-1) + m]. \quad . \quad . \quad . \quad . \quad (4a)$$

Values of $\frac{A}{p}$ and $\frac{B}{p}$ calculated for various values of ϕ and m are shown in *Table* I. Where the exact value of ϕ is not shown in the table $\frac{A}{p}$ and $\frac{B}{p}$ may be found from (3a) or (4a), or by linear interpolation. Having determined the

value of n_1 and the product $m\phi p$, the value of a_e appropriate to the quotient $\frac{Q}{c}$ is read from the graph in *Fig.* 2. The compressive stress c may then be calculated from $c = \frac{Q}{a_e}$.

FIG. 2.—In the graphs the following equations are used.

$$M = c.bd^2\left[\frac{n_1}{2}\left(1 - \frac{n_1}{3}\right) + \phi p(1 - k_1)\left(\frac{n_1 - k_1}{n_1}\right)(m - 1)\right] \quad . \quad (5)$$

$$\frac{M}{c.bd^2} = \frac{n_1 a_1}{2} + \phi p(1 - k_1)\left(\frac{n_1 - k_1}{n_1}\right)(m - 1). \quad . \quad . \quad (5a)$$

$$\frac{Q}{c} = a_e \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

$$a_e = \frac{n_1 a_1}{2} + \phi p(1 - k_1)\left(\frac{n_1 - k_1}{n_1}\right)(m - 1) \quad . \quad . \quad . \quad (7)$$

The graphs in *Fig.* 2 have been plotted for values of n_1 from $n_1 = 0{\cdot}225$ to $n_1 = 0{\cdot}5$, and for values of a_e from 0·15 to 0·6 at intervals of the product $m\phi p$ equal to 0·025 from $m\phi p = 0{\cdot}075$ to $m\phi p = 0{\cdot}45$.

The value of k_1 is 0·05, which is a useful value of the ratio of the distance from the face of the beam to the centre-line of the compressive steel and the

TABLE I.

K_1-0·05		VALUES OF ϕ 0·2	0·4	0·6	0·8	1·0
VALUES OF m 8	A/P	18·80	21·60	24·40	27·20	30·00
	B/P	64·56	65·12	65·68	66·24	66·80
10	A/P	23·60	27·20	30·80	34·40	38·00
	B/P	80·72	81·49	82·16	82·88	83·60
12	A/P	28·40	32·80	37·20	41·60	46·00
	B/P	96·88	97·76	98·64	99·52	100·40
15	A/P	35·60	41·20	46·80	52·40	58·00
	B/P	121·12	122·24	123·36	124·48	125·60
18	A/P	42·80	49·60	56·40	63·20	70·00
	B/P	145·36	146·72	148·08	149·44	150·80
20	A/P	47·60	55·20	62·80	70·40	78·00
	B/P	161·52	163·04	164·56	166·08	167·60
22	A/P	60·80	69·70	69·20	77·60	86·00
	B/P	177·68	179·36	181·04	182·72	184·40

effective depth (the "inset ratio"), as this value is suitable for many practical cases.

METHOD OF USING TABLE I AND FIG. 2.—To determine the stresses in a doubly-reinforced beam subjected to a bending moment,

(1) Determine the value of Q in $Q = \frac{M}{bd^2}$, where M is the bending moment, b the breadth of the section, and d the effective depth.

(2) Having calculated the value of p, p' and ϕ and decided the value of m:

(a) Interpolating $\frac{A}{p}$ and $\frac{B}{p}$ from *Table* I, find A and B by multiplying both by p.

(b) Determine n_1 from $n_1 = \frac{-A + \sqrt{A^2 + B}}{2}$.

(3) From *Fig.* 2, for the value of the product $m\phi p$ corresponding to the

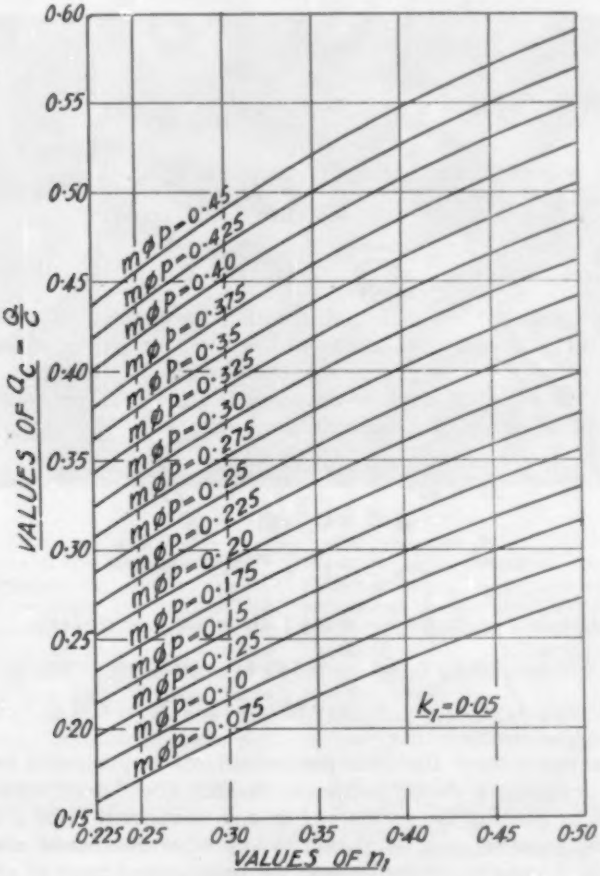

Fig. 2.

value of n_1: (*a*) Interpolate $a_e = \frac{Q}{c}$ and calculate c from $c = \frac{Q}{a_e}$. (*b*) Calculate t from $t = \frac{(1 - n_1)}{n_1} \times mc$.

EXAMPLE I.—A doubly-reinforced concrete beam is to be subjected to a bending moment of 5,330,000 in.-lb. The breadth of the beam is 22 in. and the effective depth 36 in. The "inset ratio" is 0·05. If the beam is reinforced with eight 1-in. diameter bars at the top, and ten 1-in. diameter bars at the bottom, calculate the stresses in the concrete and steel due to the bending moment.

Proceeding as described in the last paragraph,

(1) $Q = \frac{5{,}330{,}000}{22 \times 36^2} = 187.$

(2) $p = 0{\cdot}01$, $\phi = \frac{p'}{p} = 0{\cdot}8$; $m = 15$.

(*a*) $\frac{A}{p} = 52{\cdot}4$; therefore $A = 0{\cdot}524$. $\frac{B}{p} = 124{\cdot}48$, that is $B = 1{\cdot}245$.

(*b*) $$n_1 = \frac{-0{\cdot}524 + \sqrt{(0{\cdot}524)^2 + 1{\cdot}245}}{2} = 0{\cdot}355 \quad . \quad . \quad . \quad (2b)$$

(3) $m\phi p = 0{\cdot}12$. From *Fig.* 2, by interpolation, $n_1 = 0{\cdot}355$ and $a_e = 0{\cdot}255$. Therefore $c = \frac{Q}{a_e} = \frac{187}{0{\cdot}255} = 733$ lb. per square inch, $mc = 15 \times 733 = 11{,}000$ lb. per square inch, and $t = \frac{1 - 0{\cdot}355}{0{\cdot}355} \times 11{,}000 = 20{,}000$ lb. per square inch. Thus the section under consideration is within the permissible stresses.

EXAMPLE II.—Design a beam to resist a bending moment of 10,000,000 in.-lb. The effective depth is to be not greater than 42 in. and the width not greater than 22 in.; the allowable stress in the concrete c is 750 lb. per square inch and in the steel $t = 20{,}000$ lb. per square inch; m is 18.

$$Q = \frac{10{,}000{,}000}{22 \times 42^2} = 258; \quad a_e = \frac{Q}{c} = \frac{258}{750} = 0{\cdot}344;$$

$$n_1 = \frac{m}{m + \frac{t}{c}} = \frac{18}{18 + \frac{20{,}000}{750}} = 0{\cdot}404.$$

By interpolation in *Fig.* 2, $m\phi p = 0{\cdot}205$; therefore $p = 0{\cdot}01423$.

From *Table* I, $\frac{A}{p} = 63{\cdot}2$ and $\frac{B}{p} = 149{\cdot}44$; therefore from equation (2*b*) $n_1 = 0{\cdot}40$. Hence $A_c = 10{\cdot}5$ sq. in. (say fifteen $\frac{15}{16}$-in. bars) and $A_t = 13{\cdot}15$ sq. in. (say fifteen $1\frac{1}{16}$-in. bars).

These examples show that this method affords a quick and easy way of checking or designing a doubly-reinforced beam. The "inset ratio", namely 0·05, assumed in plotting the curves in *Fig.* 2, is considered to be a useful one; similar graphs, however, may be plotted to any other convenient inset ratios if desired. *Table* I and *Fig.* 2 may be used for an unlimited range of stress ratios, modular ratios, and ratios of compressive steel to tensile steel.

Printing Works for the Bank of England.

PRESTRESSED PRECAST MEMBERS.

A NEW printing works now being built at Debden, Essex, occupies a site of about 440 yd. by 200 yd. The main building is about 800 ft. long and has a maximum width of 300 ft. A second building contains the canteen and recreation and committee rooms, and is connected to the main building by a subway. The total floor space will be about 443,000 sq. ft.

Most of the main building has two floors, and as the site slopes both are partly at ground level; the upper floor is to be used as a production area and the lower floor contains the heating and electrical plant, cloakrooms, and engineers' workshops. The main production hall occupies the entire northern side of the building; south of this hall are the general printing hall, storerooms, and a smaller production hall. The southern parts of the building are three, four, and five stories high, containing offices and laboratories. At the eastern end there are also multiple-story blocks over an underground boiler-house.

Fig. 1.—The Main Hall.

The Main Hall.

The main hall (*Figs.* 1 and 2) has an arched north-light roof without internal supports. The arch has a greater curvature on the northern side in order to admit more light from the north and to give the necessary clearances for machinery. In order to accommodate ventilation, heating, electrical, and plumbing services, the two ribs comprising each arch (*Fig.* 3) are 3 ft. 6 in. apart. The hall was built in twenty-two identical sections, each about 36 ft. long and consisting of double arch-ribs with north-light shells spanning between them. The arches were prestressed by the Freyssinet system; some details of the cables are shown in *Fig.* 4. The cross section of the arches had to be as small as possible below the level of the roof of the gallery seen to the right of *Fig.* 3. The ties at the level of the upper ground floor produced high bending moments on these parts of the arches, but the prestressing of the arches reduced the force from the ties and consequently the bending moments.

The main arches of the larger production hall were precast in sections and the slabs between them cast in situ. The central sections of the arches weigh about 4 tons each, and were cast on their sides (*Fig.* 5) in order to facilitate the placing

of the ducts for the prestressing cables and the fixings for the services. A crane of 5 tons capacity with a jib 120 ft. long was used to lift the sections on to a travelling gantry of 60 tons capacity, which supported the sections in position during the prestressing operation. *Fig.* 6 shows a section of an arch ready for hoisting. The gantry was articulated so that by means of jacks at its main supports an accurate profile of the arch was achieved. The shutters for the shell roofs between the arches were in one length to span between the main towers of the gantry. These shutters were raised and lowered in the gantry frame by pulleys and winches and, when the main frame of the gantry was jacked to its correct alignment and profile, the shutters were raised into position and secured with bolts. After the concrete had hardened, the shutters were lowered by the winches. The prestressing wires in the arches were then tensioned and the gantry was lowered by the jacks on to bogies and moved to the next bay; the process of lowering and erecting the gantry required only a few hours. The lower parts of the arches and galleries were cast in situ, the large shutters used being handled by the derrick crane. The fluted front of the galleries was precast on a teak mould-base in order to ensure a good finish.

The upper ground floor is generally of flat-slab construction. Below the main hall the columns form bays 18 ft. by 19 ft.; the columns are without capitals in order to allow services to pass freely below this floor. In the case of the general printing hall, the capitals of the columns which form 24-ft. bays are formed by cantilever brackets under the floor.

Fig. 3.—Part of the Main Hall showing the Gallery.

Fig. 2.—The Main Hall from the South-east.

General Printing Hall.

The general printing hall occupies the space between the multiple-story blocks on the southern elevation and the main hall. An expansion joint is provided at each side of the roof and a balanced cantilever construction was used (*Fig.* 7) so that complete separation is obtained from the structure on either side. These beams were prestressed in order to reduce the deflection and the thickness of the beams compared with reinforced concrete.

The 5-tons derrick crane was used in the construction of this roof, which is 94 ft. wide. It consists of four free-standing bays, each comprising four cantilever beams between which there are north-light shell roofs. Each shell is about 24 ft. long, so that each bay of three shell roofs with cantilever supporting beams is about 72 ft. long. The shells were precast, and weigh about 5 tons each. The beams were cast in situ, using large shutters which were handled by the crane. The beams are 6 ft. 9 in. deep and 9 in. or 6 in. wide.

The prestressing wires in the cantilever beams were tensioned by the Gifford-Udall-CCL system; the wires are in twelve layers, each layer being longer than the one below it. The anchorage is a metal plate 1 in. thick set in the concrete; the force applied to these anchorages in the case of beams 9 in. wide was about 60,000 lb.

There are eight shells (*Fig.* 8) between each pair of beams; the shells are about 1 in. shorter than the space between the beams. The spaces between the ends of each shell and the face of the beam, in which there is a $\frac{1}{4}$ in. recess, were filled with mortar. When all the shells in a bay were in position and the joints filled with mortar, the beams and shells were prestressed together along the shells and through the beams by six wires in each

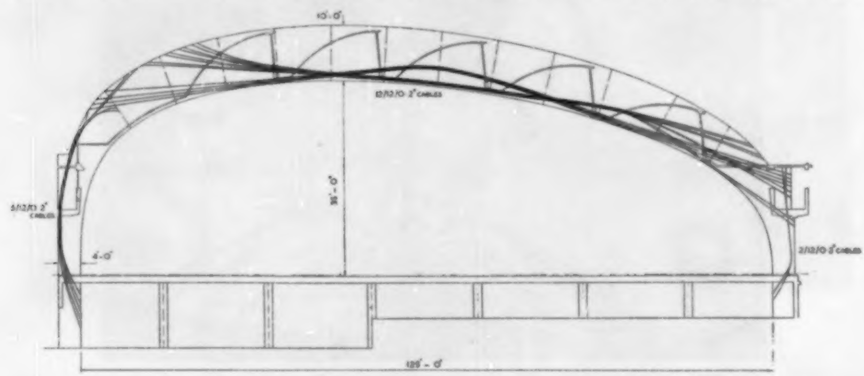

Fig. 4.—Cross Section through Main Production Hall.

Fig. 5.—Casting sections of an Arch.

shell. The beams were then prestressed by tensioning the lowest layer of wires first. A temporary load was applied to the ends of the beams, so that as the wires were tensioned the concrete in the top of the beam was not over-compressed and the concrete cover to the wires would not be cracked.

The beams were loaded in two stages, first sufficiently to ensure that the concrete was not over-compressed during the prestressing, and secondly to produce a deflection in the cantilevers before covering the wires with concrete, so that when the load was removed there would be an upward deflection of the cantilever and a consequent compression of the concrete in the top of the beam. The deflection was produced by a steel rod anchored by a stirrup to the underside of the ends of the beams and joined to a short beam, one side of which was bolted to a plate in the floor and the other side joined to a similar plate by means of single prestressing wire. In this way, by tensioning the wire in stages, and packing between the beam

Fig. 6.—Sections of an Arch.

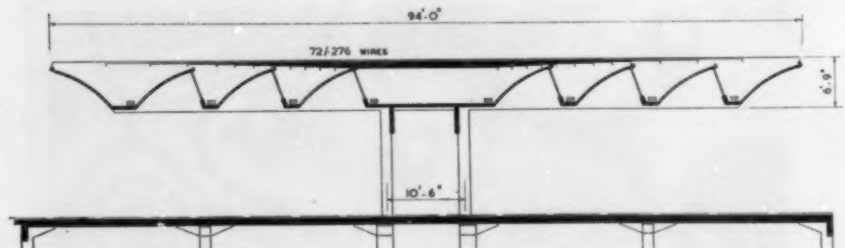

Fig. 7.—Cross Section through General Printing Hall.

and the anchor, a gradual load was applied to the steel rod and therefore to the cantilever. The magnitude of the load was measured by the gauge on the jack and by measuring the extension of the rod. A force up to about 7 tons was exerted on each cantilever.

Multiple-story Buildings.

The floors of the multiple-story buildings span between 24 ft. and 30 ft. They consist of precast beams 1 ft. 3 in. deep at 2 ft. centres covered by a 3-in. slab. These beams rest on edge-beams 2 ft. 9 in. deep supported by columns at 12 ft. or 6 ft. centres. The same size beams are used for all spans and loads, but the reinforcement is varied as required. A false ceiling suspended from the floors is used to conceal the services. The main columns are about 30 ft. high and some of them were precast so as to avoid shutter marks. The columns were cast at the site and weigh about 4½ tons each.

The precast beams forming the floors are of inverted L shape, the horizontal arm of one beam resting on a recess in the horizontal arm of the next. Where the load is heavy the beams were made continuous in a transverse direction by prestressing them together in the thickness of the floor with single wires. The beams have reinforcement projecting at each end which is cast into the beams on which they are supported. These beams also join the precast columns on the front elevations. All the floor beams (about 1600) were cast on the site.

Retaining Wall.

A retaining wall on the north of the site is designed as a cantilever. It is about 1000 ft. long and varies in height from about 3 ft. to 14 ft. It is finished with a vertical corrugated pattern, and since it had to be constructed in a trench containing many struts it was necessary to provide holes through the wall; these holes are arranged in a regular pattern as they would be prominent in the finished work. The greater part of the wall was precast in units about 9 ft. long weighing up to about 5 tons in order that the holes could be spaced at regular intervals and that a good finish to the surface could be obtained. The precast units were lowered into position through the timbers of the trench, and the junction between the lower part of the wall and the foundation was formed in situ.

The foundations are generally isolated bases and rest on brown clay which, at 7 ft. below ground level, is capable of sustaining a load of 2 tons per square foot. Other work below ground consists of ducts of various sizes for services. The ducts are in reinforced concrete; the walls were precast in sections about 10 ft. long by generally 6 ft. 6 in. high, and a joint 12 in. wide between the sections was formed in situ.

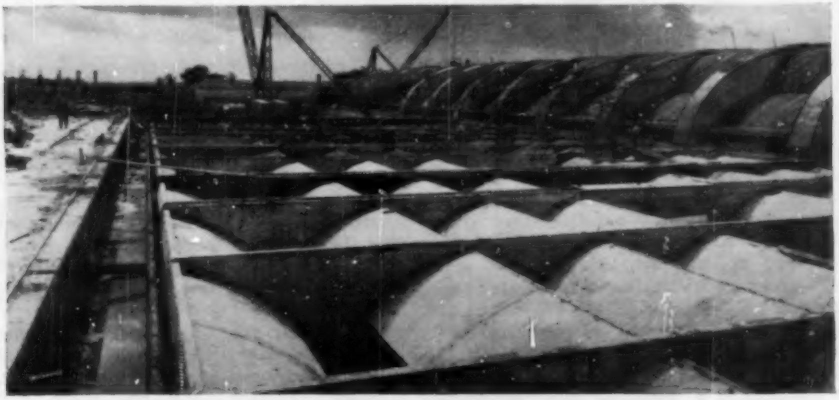

Fig. 8.—Roof of the General Printing Hall.

Concrete Plant.

Since about fifteen different concrete mixtures were used, all of which were to be vibrated, a central mixing plant was used to produce most of the concrete, with two smaller mixing plants at remoter parts of the site. The central plant had four aggregate hoppers and two cement hoppers for different types of cement. This mixer discharged into weighing-hoppers and thence into an electrically-driven loading hopper, and was controlled by one man. There were two 1 cu.-yd. mixers, one supplying skips travelling on 2-ft. gauge track and the other supplying a pump. The concrete for the more massive foundations and machine floors was pumped for long distances, while the concrete transported in skips was hoisted by crane and placed in the roofs and columns.

Shuttering.

The shuttering was designed and made on the site. To give a good finish, resin-bonded plywood was widely used. Nearly all the in-situ concrete was cast in steel shuttering. For the floors of the main machinery halls, which are from 1 ft. to 1 ft. 3 in. thick, the shutters were erected on a framework that could travel and be moved as the work proceeded.

The architects are Messrs. Easton & Robertson, the consulting engineers Messrs. Ove Arup & Partners, and the contractors Sir Robert McAlpine & Sons, Ltd.

The Engineer in Society.

[We are pleased to publish the following letter from Mr. C. A. Risbridger, B.Sc., M.Inst.C.E., Chief Engineer of the City of Birmingham Water Department. It emphasises our views on the folly of scientists and engineers saying stupid things, and of publicity being given to such statements. We fully agree that engineers and their works should be properly appreciated, but this will not be achieved by talking nonsense and making ridiculous claims. We made no suggestion that scientists were responsible for the policy of those who direct the use of new discoveries.]

" Sir,—It is a welcome and refreshing departure from practice when an Editorial article in a technical publication is devoted to a socialogical subject, as was the case in your October issue. It would be equally welcome and refreshing were you to permit another departure from custom by including in your next issue a ' Letters to the Editor ' column[*], for the article to which I have referred contains statements which, left unqualified, would leave the reader with a completely false impression that scientists in general and engineers in particular are as devoid of wisdom as they are rich in self-conceit.

" It is to be regretted that your article did not point out that the quotations from the Journal of the Engineers' Guild were from a letter to the Editor, which, whilst in the main not at all flattering to engineers in general, was decorated here and there with gems of auto-suggestion as those which you quoted.

" Time was, Sir, when the engineer was content to do, and to leave to others the writing and the speech-making. That has led to the unfortunate and false impression that solely because a man is an engineer he is not competent to be a ' leader of men or an adviser on everyday affairs ', to quote you. Your statement that there are undoubtedly leaders of men among engineers shows that you do not subscribe to that view. You have, however, subscribed to a very common injustice when you suggest that because scientists have made possible the internal combustion engine, bombing, poison gas, bacterial warfare, and the atom and hydrogen bombs they are also responsible for the misuse of their knowledge. It is not the scientists who direct the use which shall be made of their discoveries but just those ' leaders of men and advisers on everyday matters ', those policy makers who so often with faint contempt refer to the scientists as ' technicians ', a lower order of life altogether.

" It is well now and again to remind ourselves that not only did the scientist make possible those things which the policy makers have misused, but that he also made possible the many things which make living more pleasant. The policy makers cannot have it both ways, accepting the credit for the proper use of the scientist's knowledge and skill, and deflecting to the scientist the discredit for their improper use."

[*We are always pleased to receive letters for publication.—Ed.]

Testing the Strength of Concrete by the Ultrasonic-pulse Method.

By R. JONES,* B.Sc., Ph.D., and J. H. WETTERN,† A.M.Inst.T.

THIS article describes the use of the ultrasonic-pulse method to assess the strength of precast units to form sections of primary and secondary beams which were to be assembled and prestressed on the site to form the beams in a new school building. The tests were undertaken at the request of the Ministry of Education and with the co-operation of Messrs. Gilbert-Ash, Ltd., the Prestressed Concrete Co., Ltd., and the Mono Concrete Co., Ltd. The testing was done at the works where the units were made, and 10 per cent. of the total number produced were

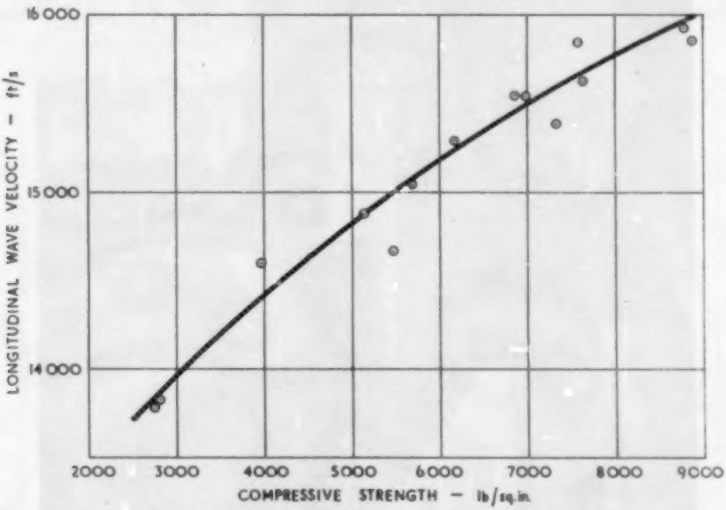

Fig. 1.—Relation between Longitudinal Wave Velocity and Compressive Strength.

tested. By this means a stricter control of quality was achieved than would have been possible by usual methods of testing.

The Ultrasonic-pulse Method.

For any concrete there is a simple relationship between the velocity of an ultrasonic pulse in the concrete and its compressive strength [1] [2]. Therefore, if it is required to know the compressive strength, the procedure is to determine the velocity of an ultrasonic pulse in the concrete and to infer the corresponding compressive strength from ancillary data relating the compressive strengths and pulse velocities of cubes made of concrete of the same ingredients and composition. These data are obtained by testing cubes at different ages. *Fig.* 1 shows this relationship for the concrete used in making the units which were the subject of the tests described. By testing cubes at different ages and made from

* Road Research Laboratory, Department of Scientific and Industrial Research.
† The Mono Concrete Co., Ltd.

different batches of concrete, allowance is automatically made for slight variations in proportions which may occur from day to day. If, however, the average proportions of the mixtures are changed, or another type of aggregate is used, a further series of test cubes would be required [1] [2].

Method of Testing.

Fig. 2 shows the testing apparatus (which is fully described elsewhere [3]) together with precast units undergoing test. The velocity of the ultrasonic pulse, that is the longitudinal wave velocity, is calculated from the time taken

Fig. 2.—Testing the Precast Concrete Units.

for the leading edge of the pulse to traverse a known length of concrete. The pulse is sent out 50 times per second from a piezoelectric-crystal transducer held in contact with one face of the concrete. After passing through the concrete, the pulse is received by a second crystal transducer which converts the mechanical pulse into a corresponding electrical signal. This is amplified and produces a visual image of the pulse on the trace of a cathode-ray tube. The image passes across the screen in synchronism with the transmitted pulse. Timing-marks also appear on the trace at intervals of ten microseconds. The time of propagation of the pulse is measured by the displacement of the pulse when the transducers are respectively (1) in contact with each other and (2) separated by the concrete. An interpolation device enables the time to be measured to $\pm 0{\cdot}1$ microsecond, and reading is facilitated by expanding the part of the time-scale

in which the signal lies. The thickness of the concrete at each measuring position is determined with the aid of calipers.

The units (*Fig.* 2) were 3 ft. 3 in. long and $7\frac{1}{16}$ in. wide across the bottom flange. The concrete in the bottom flange required the most careful examination since this part sustains the highest initial stresses. Analysis of a large number of measurements showed that the mean result from three standard positions was within ± 1 per cent. of the true mean time of transmission. A standard procedure of test was therefore adopted, consisting of taking three measurements directly across the $7\frac{1}{16}$-in. dimension (one measurement near each end and one at the middle).

Initially it was desired that every unit be tested, but as a result of extensive preliminary measurements it was found that sampling methods could be employed. The decision to test 10 per cent. of all the units produced proved satisfactory both from a practical and from an experimental point of view.

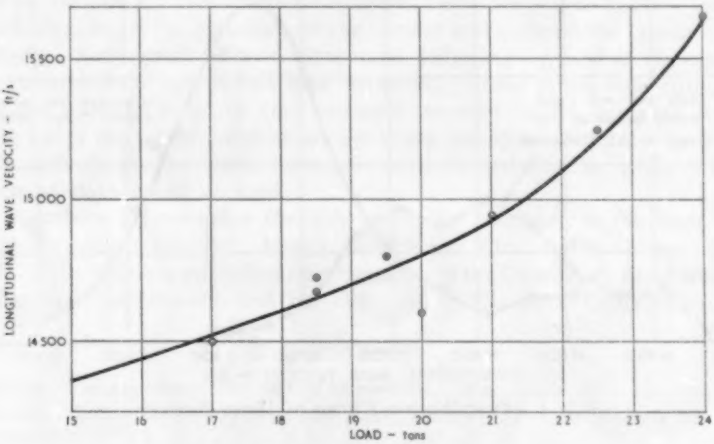

Fig. 3.—Results of Compressive Tests.

In order to keep handling costs low the testing apparatus had to be as near as possible to the stacking area, and the instrument was therefore housed in a small weatherproof building close to the casting shop. Access was by double doors on opposite sides of the building, which enabled a roller conveyor carrying the units to pass through. The apparatus was operated by the staff of the Mono Concrete Co., Ltd., and Messrs. Gilbert-Ash, Ltd., and the testing was fitted into normal factory routine.

One unit from each batch was set aside when it was removed from the mould and placed close to the testing machine. As the units attained the necessary age they were prepared for testing in batches of about a dozen. Preparation consisted of numbering the units for record purposes, cleaning, and applying soap-jelly to the surface in the positions at which the transducers were to be applied. Particles of grit and air bubbles, if allowed to come between the transducers and the concrete, tend to cause distortion of the cathode-ray trace; carborundum stone and a clean rag ensure the necessary smooth surface. The

soap-jelly provides a coupling between the transducer and the concrete ; this was as effective as the oil normally used and did not stain the concrete.

Before each day's testing a measurement was made on an aluminium proving-block through which the time of transmission of the ultrasonic pulse was accurately known. This enabled the operator to ensure that the apparatus was working satisfactorily. *Fig.* 2 shows a test in progress. An assistant holds the transducers in firm contact with the concrete while the operator observes the cathode-ray trace and records the reading. The rate of progress was governed more by the time required for handling and preparing the units than by the time taken in actual testing. The average rate was about twenty units per hour.

The specification required the cube strength of the concrete to be at least 6500 lb. per square inch at 28 days. The first measurements of pulse velocity showed that some of the units would have a somewhat lower equivalent cube

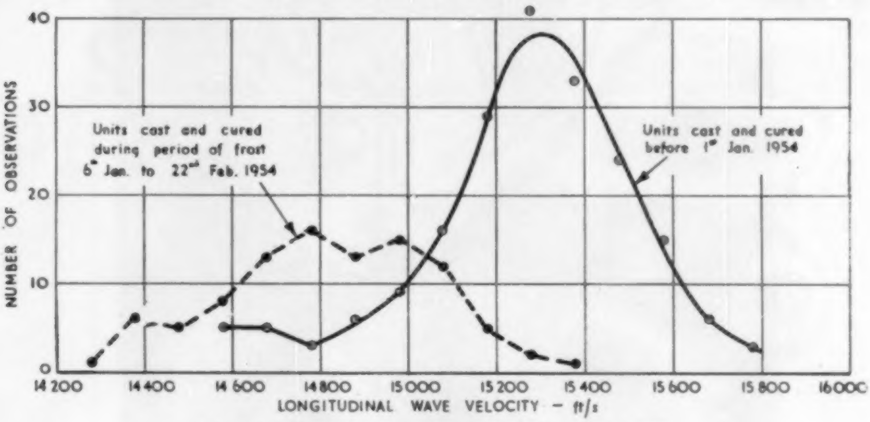

Fig. 4.—Distribution Curve of Mean Results.

strength at this age. The units were designed for a total load of 18 tons on the flange, including the factor of safety. Compression tests showed that units failing under this load would have strengths corresponding to a pulse velocity in the flange of 14,600 ft. per second (*Fig.* 3). To provide a slight additional margin, 14,800 ft. per second, corresponding to an equivalent cube strength of 5000 lb. per square inch, was chosen as the acceptance figure.

Test Results.

The first 200 units tested were cast and cured between July and December, 1953, when the weather was relatively mild. Subsequent units were cast and cured in January, 1954, when severe frosts occurred. The apparatus was not available until late in November, 1953, so that some of the first 200 units were considerably over-age at the time of test. The results from these units were adjusted to relate to an age of 28 days by using additional data obtained from ageing tests on cubes.

The distribution of the longitudinal wave velocities in the units tested before 1 January, 1954, is given in *Fig.* 4. The results are on a normal distribu-

tion curve except for ten units which gave abnormally low values between 14,500 and 14,700 ft. per second. These were all cast, although not necessarily tested, on the same day. For the normal part of the distribution the mean value of the equivalent cube strength was 6300 lb. per square inch and the standard deviation ±500 lb. per square inch. Analysis of results obtained on cubes during the same period gave a mean strength of 7750 lb. per square inch and a standard deviation of ±750 lb. per square inch. As expected, the strength of the concrete in the flange of the unit was slightly inferior to that in the corresponding test cube, but generally much higher than the acceptance strength. The variability, however, was similar and it is concluded that the main fluctuations in strength were due to variations between the different batches of concrete.

Results obtained from a further 97 units cast and cured during the intermittent frosts of January and February, 1954, are also given in *Fig.* 4. The strength of this concrete was appreciably less than that of the previous batch and there was wider variation. The difference is almost certainly due to the low temperatures, which delayed the hydration of the cement and reduced the rate of increase of strength. Subsequent tests on these units, following a period of thaw, showed when the required strength had been attained. Under favourable curing conditions the specified strength (the strength required before applying the prestressing force) can be attained at an age much less than 28 days. Thus, when an early delivery was required, ultrasonic tests showed whether units of an age less than 28 days could be used.

The authors acknowledge the help given by members of the staff of the Prestressed Concrete Co., Ltd., Messrs. Gilbert-Ash, Ltd., and the Mono Concrete Co., Ltd. The article is published by permission of the Director of Road Research, Road Research Laboratory, and the Directors of the Mono Concrete Co., Ltd.

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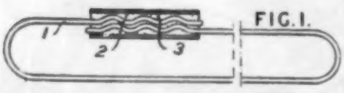
(2) JONES, R. Testing concrete by ultrasonic pulse technique. *Proc. Highw. Res. Bd., Wash.*, 1953, **32**, 258–75.

(3) GATFIELD, E. N. An apparatus for determining the velocity of an ultrasonic pulse. *Electron. Engng.*, 1952, **24**, (295) 390–5.

Patented Method of Joining Reinforcement.



A JOINT for connecting reinforcement (1) consists of a tubular element (2) which encloses parts of the bars projecting into or passed through the element and is filled besides the bars with compacted sand or like granular material (3). The ends of the bars may be straight or deformed (*Fig.* 1). The sand (3) is compacted in the tube by vibrating or shaking the tube. A loop as shown may consist of a plurality of windings in which case more than two bars are passed through the tube (2). A joint may be made between the ends of the bars.—British Patent No. 678,543. Concrete Patents, Ltd., and K. R. Danhof, October 18, 1950.

Reinforced Concrete Chimney 615 ft. High.

THE chimney illustrated in *Figs.* 1 and 2 is being built at Copper Cliff, Ontario, Canada, for the International Nickel Company of Canada, Ltd. The external diameters are 63 ft. 6 in. at the base and 33 ft. at the top; the internal diameters are 55 ft. 4 in. at the base and 30 ft. at the top. The base is 22 ft. high. The weight of the stack will be 17,000 tons, and the total volume of concrete 7400 cu. yd., including 1635 cu. yd. in the base. The reinforced concrete shaft is insulated with bricks.

The working platform is formed of steel and aluminium tubes; it weighs 16 tons, which is estimated to be 10 tons less than a wooden platform. The moving shutters are of steel. The platform is raised 7 ft. at a time after each 7-ft. lift of concrete is placed. The concrete is raised in buckets of ½ cu. yd. capacity by a 2-tons hoist and tipped into a hopper on the platform, from which it is distributed to the shutters by barrows containing 1½ cu. ft. The chimney was designed and is being built by the Custodis Company.

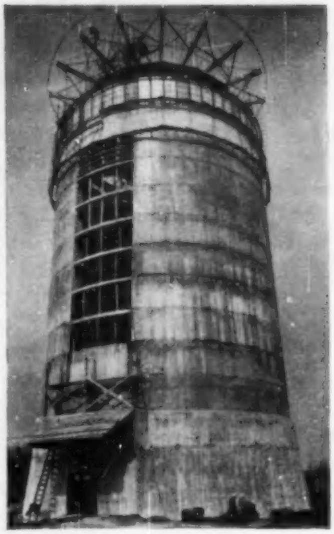


Fig. 1.—The Base.

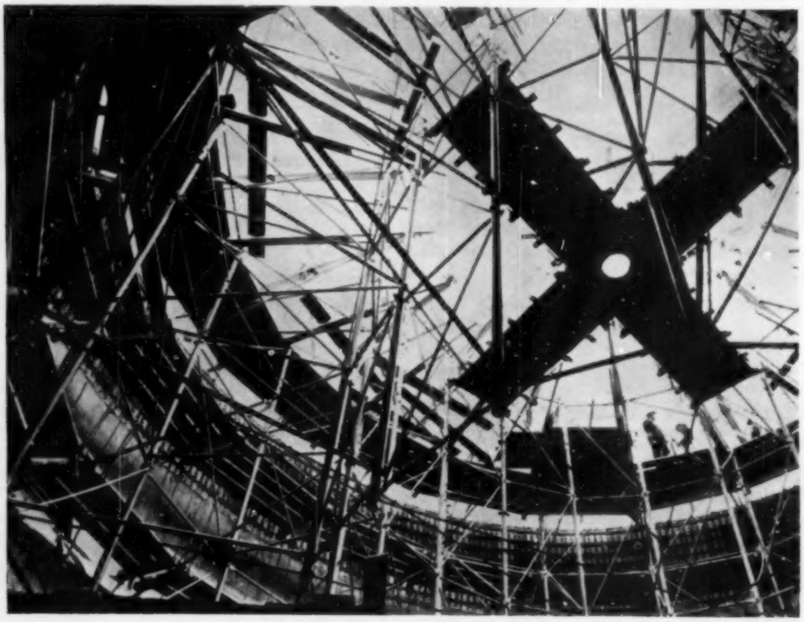


Fig. 2.—The Working Platform.

Analysis of Statically-indeterminate Structures by the Deformation Method.—V.*

By M. SMOLIRA, Ph.D., A.M.Inst.C.E., D.I.C.

Multiple-span Frames with Curved Members.

In a similar way, equations of equilibrium may be set out for frames with any number of spans. An increase in the number of statically-indeterminate bending moments is accompanied by a similar increase in the number of equations. For symmetrical frames and loading, however, the number of equations of equilibrium can be reduced by half.

For a four-span symmetrical frame with symmetrical loading (*Fig.* 45),

$$\left.\begin{aligned}
&DAB \ldots m_1.\alpha_{ab} + m_1.\alpha_{ad} + m_2.\beta_1 + H_d.\gamma_1 && = \theta_{ab} + \frac{\Delta a}{h}\\
&ABC \ldots m_1.\beta_1 + m_2\alpha_{ba} + m_3\alpha_{bc} + m_4.\beta_2 + H_d.\delta_1 + (H_d + H_e).\gamma_2 && = \theta_{ba} + \theta_{bc}\\
&EBC \ldots m_3.\alpha_{bc} + (m_3 - m_2)\alpha_{be} + m_4.\beta_2 + m_3\frac{\Delta_2^m}{h} + m_4\frac{\Delta_2^m}{h} + \\
&\qquad\qquad + (H_d + H_e).\gamma_2 + (H_d + H_e).\frac{\Delta_2^h}{h} && = \theta_{bc} + \frac{\Delta_2^o}{h}\\
&B \ldots m_1.\Delta_1^m + m_2.\Delta_1^m + m_3\Delta_2^m + m_4.\Delta_2^m + H_d.\Delta_1^h + (H_d + H_e).\Delta_2^h && = \Delta_1^o + \Delta_2^o - \Delta a\\
&CB \ldots m_4.\alpha_{cb} + m_3.\beta_2 + (H_d + H_e).\delta_2 && = \theta_{cb}
\end{aligned}\right\} \quad (70a)$$

If the second and third spans are loaded symmetrically:

$$\left.\begin{aligned}
&DAB \ldots m_1.\alpha_{ab} + m_1.\alpha_{ad} - m_2.\beta_1 + H_d.\gamma_1 && = \frac{\Delta a}{h}\\
&ABC \ldots m_1.\beta_1 - m_2.\alpha_{ba} + m_3\alpha_{bc} + m_4.\beta_2 + H_d.\delta_1 + (H_d + H_e).\gamma_2 && = \theta_{bc}\\
&EBC \ldots m_3\alpha_{bc} + (m_2 + m_3).\alpha_{be} + m_4.\beta_2 + m_3.\frac{\Delta_2^m}{h} + m_4.\frac{\Delta_2^m}{h} + \\
&\qquad\qquad + (H_d + H_e).\gamma_2 + (H_d + H_e)\frac{\Delta_2^h}{h} && = \theta_{bc} + \frac{\Delta_2^o}{h}\\
&B \ldots m_1\Delta_1^m - m_2\Delta_1^m + m_3\Delta_2^m + m_4.\Delta_2^m + H_d.\Delta_1^h + (H_d + H_e).\Delta_2^h && = \Delta_2^o - \Delta a\\
&CB \ldots m_4\alpha_{cb} + m_3.\beta_2 + (H_d + H_e).\delta_2 && = \theta_{cb}
\end{aligned}\right\} \quad (70b)$$

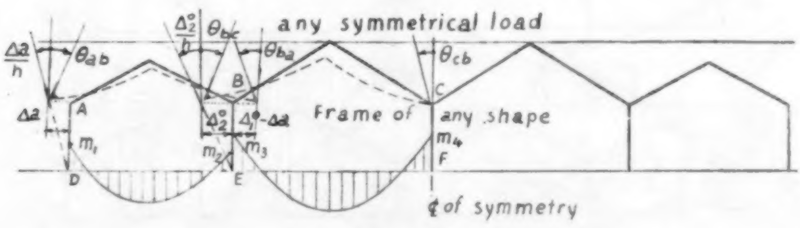


Fig. 45.

* Previous articles appeared in this Journal for July, August, September and October, 1954.

For symmetrical frames with unsymmetrical loading (*Fig.* 46*a*), the number of simultaneous equations may also be reduced by half if the actual loading is replaced by two equivalent systems of loading, namely, one symmetrical (*Fig.* 46*b*), and one unsymmetrical (*Fig.* 46*c*). The final result is obtained by combining the results of these two equivalent cases.

For a four-span symmetrical frame with an unsymmetrical load on the second span only (*Fig.* 46*a*), the two equivalent loadings are as shown in *Figs.* 46*b* and 46*c*. Equations (70*b*) apply to a symmetrical system of loading. For unsymmetrical loading the equations of equilibrium are

$$\left.\begin{array}{ll}
DAB \ldots & m_1 . \alpha_{ab} + m_1 . \alpha_{ad} - m_2 . \beta_1 + H_d \, \gamma_1 = \frac{\Delta a}{h} \\
ABC \ldots & m_1 . \beta_1 \; - m_2 \alpha_{ba} + m_3 . \alpha_{bc} + m_4 . \beta_2 + H_d . \delta_1 + (H_d + H_e) . \gamma_2 = \theta_{bc} \\
EBC \ldots & m_3 . \alpha_{bc} + (m_2 + m_3) . \alpha_{be} + m_4 . \beta_2 + m_3 . \frac{\overset{m}{\Delta}_2}{h} + m_4 . \frac{\overset{m}{\Delta}_2}{h} + \\
 & \qquad + (H_d + H_e) . \gamma_2 + (H_d + H_e) . \frac{\overset{h}{\Delta}_2}{h} = \theta_{bc} + \frac{\Delta_2^{\circ}}{h} \\
B \ldots & m_1 . \overset{m}{\Delta}_1 - m_2 . \overset{m}{\Delta}_1 + m_3 . \overset{m}{\Delta}_2 + m_4 . \overset{m}{\Delta}_2 + H_d . \overset{h}{\Delta}_1 + (H_d + H_e) \overset{h}{\Delta}_2 = -\Delta_2^{\circ} - \Delta a \\
BCF \ldots & m_4 . \alpha_{cb} + 2 m_4 . \alpha_{cf} + m_3 . \beta_2 + (H_d + H_e) . \delta_2 = \theta_{cb}
\end{array}\right\} . \quad (71)$$

EXAMPLE.—The elastic constants and load functions for the frame shown in *Fig.* 47 are:

For $m_{ab} = 1$: $EI\alpha_{ab} = 16{\cdot}64$; $EI\beta = 6{\cdot}17$; $EI\Delta^m = 155{\cdot}67$.

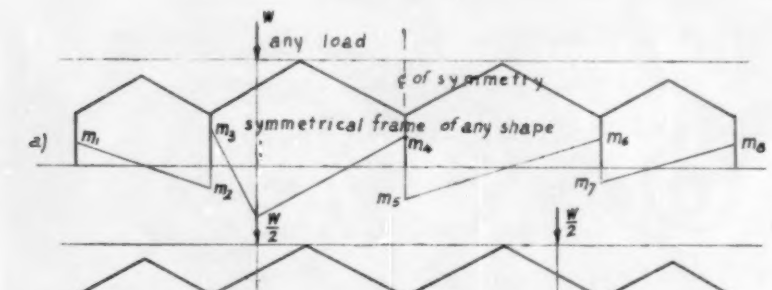


Fig. 46.

For $m_{ba} = 1$: $EI\alpha_{ba} = 11{\cdot}27$; $EI\beta = 6{\cdot}17$; $EI\Delta^m = 85{\cdot}8$.

For $H = 1$: $EI\gamma = 155{\cdot}67$; $EI\delta = 85{\cdot}8$; $EI\Delta^h = 1932$.

For a uniformly distributed load:

$EI\theta_{ab} = 876$, $EI\theta_{ba} = 1314$, and $EI\Delta^o = 17{,}000$, and, from (67),

$$\text{EAB} \ldots 16{\cdot}64m_1 + \tfrac{12}{3}m_1 + 6{\cdot}17m_2 + \tfrac{85{\cdot}8}{12}m_2 + \tfrac{155{\cdot}67}{12}m_1 + \tfrac{155{\cdot}67}{12}m_1 + \tfrac{1932}{12^2}m_1 = 876 + \tfrac{17{,}000}{12} + \tfrac{\Delta b}{h}EI$$

$$\text{ABC} \ldots 6{\cdot}17m_1 + 11{\cdot}27m_2 + 16{\cdot}64m_3 + 6{\cdot}17m_4 + \tfrac{85{\cdot}8}{12}m_1 + \tfrac{155{\cdot}67}{12}(m_1 + m_3 - m_2) = 1314 + 876$$

$$\text{FBC} \ldots 16{\cdot}64m_3 + \tfrac{12}{3}(m_3 - m_2) + 6{\cdot}17m_4 + \tfrac{155{\cdot}67}{12}(m_1 + m_3 - m_2) = 1314 + \tfrac{\Delta b}{h}EI$$

$$\text{C} \ldots \tfrac{155{\cdot}67}{12}m_3 + \tfrac{85{\cdot}8}{12}m_4 + \tfrac{1932}{12^2}(m_1 + m_3 - m_2) = \tfrac{17{,}000}{12} - \tfrac{\Delta b}{h}EI - \tfrac{\Delta c}{h}EI$$

$$\text{BCD} \ldots 6{\cdot}17m_3 + 11{\cdot}27m_4 + 16{\cdot}64m_5 + 6{\cdot}17m_6 + \tfrac{85{\cdot}8}{12}(m_1 + m_3 - m_2) + \tfrac{155{\cdot}67}{12}m_6 = 1314 + 876$$

$$\text{BCG} \ldots 6{\cdot}17m_3 + 11{\cdot}27m_4 + \tfrac{12}{3}(m_4 - m_5) + \tfrac{85{\cdot}8}{12}(m_1 + m_3 - m_2) = 1314 + \tfrac{\Delta c}{h}EI$$

$$\text{CDK} \ldots 11{\cdot}27m_6 + \tfrac{12}{3}m_6 + 6{\cdot}17m_5 + \tfrac{155{\cdot}67}{12}m_5 + \tfrac{85{\cdot}8}{12}m_6 + \tfrac{85{\cdot}8}{12}m_6 + \tfrac{1932}{12^2}m_6 = 1314 + \tfrac{17{,}000}{12} + \tfrac{\Delta c}{h}EI$$

$$\text{S.c} \ldots m_1 - m_2 + m_3 = m_4 - m_5 + m_6$$

from which $m_1 = 40{,}559{\cdot}1$ ft.-lb., $m_2 = 11{,}293{\cdot}9$ ft.-lb., $m_3 = 32{,}191{\cdot}6$ ft.-lb., $m_4 = 30{,}732{\cdot}8$ ft.-lb., $m_5 = 17{,}286{\cdot}5$ ft.-lb., and $m_6 = 48{,}010{\cdot}5$ ft.-lb.

EXAMPLE.—The elastic constants and load functions for the frame shown in *Fig.* 48 calculated by the method of summation are:

For $m_{ab} = 1$: $EI\alpha = 16{\cdot}01$; $EI\beta = 6{\cdot}10$; $EI\Delta^m = 171{\cdot}78$.

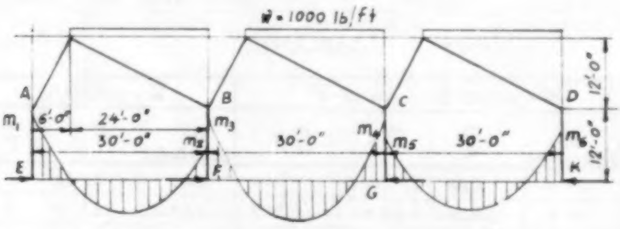


Fig. 47.

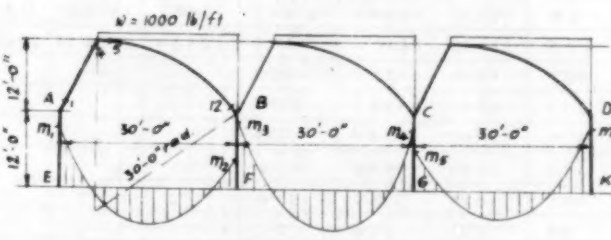


Fig. 48.

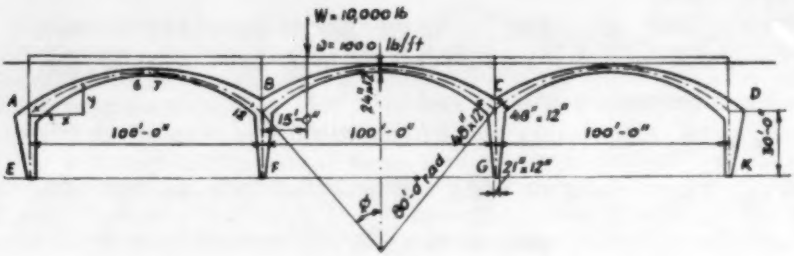


Fig. 49.

TABLE II.

Pt.	x ft	y ft	a ft	I ft⁴	$m_a = 1$	$\frac{m}{I}$	$\frac{m}{I} \cdot x$	$\frac{m}{I} y$	$\frac{y^2}{I}$
1	3·65	2·74	4·50	7·59	0·975	0·1285	0·469	0·352	0·99
2	10·26	7·55	3·60	3·89	0·923	0·2375	2·437	1·793	14·65
3	19·33	11·45	3·04	2·34	0·872	0·3720	7·191	4·259	56·03
4	27·90	14·18	2·52	1·34	0·818	0·6110	17·047	8·664	150·05
5	36·65	16·12	2·20	0·89	0·726	0·8180	29·979	13·186	292·96
6	45·50	17·12	2·05	0·72	0·612	0·8510	38·720	14·569	407·07
7	54·50	17·12	2·05	0·72	0·502	0·6970	37·986	11·932	
8	63·35	16·12	2·20	0·89	0·396	0·4470	28·317	7·206	
9	72·10	14·18	2·52	1·34	0·296	0·2210	15·934	3·134	
10	80·67	11·45	3·04	2·34	0·200	0·0855	6·897	0·979	
11	89·74	7·55	3·60	3·89	0·112	0·0288	2·584	0·217	
12	96·35	2·74	4·50	7·59	0·035	0·0046	0·443	0·012	
Σ	—	—	—	—	—	4·5019	188·007	66·305	930·66

TABLE III.

	W = 10,000 lb. a = 15'-0".				w = 1000 lb/ft.		
Pt.	m_0 (1000 lb.-ft.)	$\frac{m}{I}$	$\frac{m}{I} \cdot x$	$\frac{m}{I} \cdot y$	m_0 (1000 lb.-ft.)	$\frac{m}{I}$	$\frac{m}{I} \cdot y$
1	29·5	3·88	14·2	10·6	245·0	32·28	88·0
2	90·5	23·26	238·7	175·7	650·0	167·10	1261·0
3	155·0	66·24	1280·4	758·4	910·0	388·89	4453·0
4	137·2	112·39	2856·6	1451·9	1100·0	820·90	11,641·0
5	112·8	127·17	4660·8	2049·9	1200·0	1352·87	21,808·0
6	87·5	121·53	5529·5	2080·6	1248·0	1733·35	29,675·0
7	68·5	95·14	5185·1	1628·8			
8	55·0	62·01	3928·1	999·6			
9	41·0	30·60	2206·0	432·9			
10	29·0	12·39	999·8	141·9			
11	16·2	4·16	373·7	31·4			
12	5·5	0·73	69·8	2·0			
Σ	—	649·50	27,342·7	9,764·6	—	4495·37	68,926·0

For $m_{ba} = 1$: $EI\alpha = 12{\cdot}91$; $EI\beta = 6{\cdot}10$; $EI\Delta^m = 128{\cdot}39$.
For $H = 1$: $EI\gamma = 171{\cdot}78$; $EI\delta = 128{\cdot}39$; $EI\Delta^h = 2729{\cdot}57$.
For *u.d.l.*: $EI\theta_a = 1205{\cdot}21$; $EI\theta_b = 1190{\cdot}84$; $EI\Delta^o = 21{,}518{\cdot}91$.

From (67),

$$\text{EAB} \ldots 16{\cdot}01\,m_1 + \frac{12}{3}m_1 + 6{\cdot}10\,m_2 + \frac{128{\cdot}39}{12}m_2 + \frac{171{\cdot}78}{12}m_1 + \frac{171{\cdot}78}{12}m_1 + \frac{2729{\cdot}5}{12^2}m_1 = 1205{\cdot}21 + \frac{21{,}518{\cdot}91}{12} + \frac{\Delta b}{h}EI$$

$$\text{ABC} \ldots 6{\cdot}10\,m_1 + 12{\cdot}91\,m_2 + 16{\cdot}01\,m_3 + 6{\cdot}10\,m_4 + \frac{128{\cdot}39}{12}m_1 + \frac{171{\cdot}78}{12}(m_1 + m_3 - m_2) = 1190{\cdot}84 + 1205{\cdot}21$$

$$\text{FBC} \ldots 16{\cdot}01\,m_3 + \frac{12}{3}(m_3 - m_2') + 6{\cdot}10\,m_4 + \frac{171{\cdot}78}{12}(m_1 + m_3 - m_2) = 1205{\cdot}21 + \frac{\Delta b}{h}EI$$

$$\text{C} \ldots \frac{171{\cdot}78}{12}m_3 + \frac{128{\cdot}39}{12}m_4 + \frac{2729{\cdot}57}{12^2}(m_1 + m_3 - m_2) = \frac{21{,}518{\cdot}91}{12} - \frac{\Delta b}{h}EI - \frac{\Delta c}{h}EI$$

$$\text{BCD} \ldots 6{\cdot}10\,m_3 + 12{\cdot}91\,m_4 + 16{\cdot}01\,m_5 + 6{\cdot}10\,m_6 + \frac{128{\cdot}39}{12}(m_1 + m_3 - m_2) + \frac{171{\cdot}78}{12}m_6 = 1190{\cdot}84 + 1205{\cdot}21$$

$$\text{BCG} \ldots 6{\cdot}10\,m_3 + 12{\cdot}91\,m_4 + \frac{12}{4}(m_4 - m_5) + \frac{128{\cdot}39}{12}(m_1 + m_3 - m_2) = 1205{\cdot}21 + \frac{\Delta c}{h}EI$$

$$\text{CDK} \ldots 12{\cdot}91\,m_6 + \frac{12}{3}m_6 + 6{\cdot}10\,m_5 + \frac{171{\cdot}78}{12}m_5 + \frac{128{\cdot}39}{12}m_6 + \frac{128{\cdot}39}{12}m_6 + \frac{2729{\cdot}5}{12^2}m_6 = 1190{\cdot}84 + \frac{21{,}518{\cdot}91}{12} + \frac{\Delta c}{h}EI$$

$$\text{S.c.} \ldots m_1 + m_3 - m_2 = m_4 - m_5 + m_6$$

from which $m_1 = 43{,}919{\cdot}1$ ft.-lb., $m_2 = 24{,}050{\cdot}9$ ft.-lb., $m_3 = 32{,}011{\cdot}6$ ft.-lb., $m_4 = 10{,}973{\cdot}2$ ft.-lb., $m_5 = 6883{\cdot}1$ ft.-lb., and $m_6 = 47{,}789{\cdot}7$ ft.-lb.

EXAMPLE.—The elastic constants and load functions for the frame shown in *Fig.* 49 are calculated by the method of summations (see *Tables* II, III and IV).

$$\sin\phi = \frac{50}{80} = 0{\cdot}625\,;\quad \phi = 38^\circ\,41'\,(0{\cdot}67516^r)\,;\quad ds = \frac{80 \times 0{\cdot}67516}{6} = 9 \text{ ft.}$$

$$\text{For } m_a = 1:\quad E\varepsilon = 4{\cdot}5019 \times 9 = 40{\cdot}52\,;\quad \bar{x} = \frac{188{\cdot}007}{4{\cdot}5019} = 41{\cdot}76 \text{ ft.};$$

$$E\alpha = 23{\cdot}6\,;\quad E\beta = 16{\cdot}92\,;\quad E\Delta^m = 66{\cdot}305 \times 9 = 596{\cdot}74.$$

TABLE IV.

Pt.	y ft	d ft	I ft⁴	m=1	$\frac{m}{I}ds$	$\frac{m}{I}y\,ds$
1	1·5	3·9	4·94	0·95	0·5765	0·86
2	4·5	3·7	4·22	0·85	0·6041	2·72
3	7·5	3·5	3·57	0·75	0·6297	4·72
4	10·5	3·3	2·99	0·65	0·6511	6·84
5	13·5	3·1	2·48	0·55	0·6647	8·97
6	16·5	2·9	2·03	0·45	0·6642	10·96
7	19·5	2·7	1·64	0·35	0·6401	12·48
8	22·5	2·5	1·30	0·25	0·5760	12·96
9	25·2	2·3	1·01	0·15	0·4438	11·32
10	28·5	2·1	0·77	0·05	0·1948	5·55
Σ	—	—	—	—	5·6450	77·38

For $H = 1$: $E\gamma = E\delta = 596{\cdot}74$; $E\Delta^h = 930{\cdot}66 \times 9 = 8375{\cdot}9$.

For $W = 10{,}000$ lb. at $a = 15$ ft. from B:

$$E\varepsilon = 649{\cdot}5 \times 9 = 5845{\cdot}5\,; \quad \bar{x} = \frac{27{,}342{\cdot}7}{649{\cdot}5} = 42{\cdot}1 \text{ ft.}$$

$$E\theta_b = 3384{\cdot}5\,; \quad E\theta_c = 2461\,; \quad E\Delta^o = 9764{\cdot}6 \times 9 = 87{,}881{\cdot}9.$$

For *u.d.l.* $w = 1000$ lb. per foot:

$$E\theta^o = 4495{\cdot}37 \times 9 = 40{,}458\,; \quad E\Delta^o = 68{,}926 \times 9 \times 2 = 1{,}240{,}668.$$

For a uniformly-distributed load, from (67):

$$\text{EAB}\ldots 23{\cdot}60m_1 + 3{\cdot}06m_1 + \frac{596{\cdot}74}{30}m_1 + \frac{596{\cdot}74}{30}m_1 + \frac{8375{\cdot}9}{30^2}m_1 - 16{\cdot}92m_2 - \frac{596{\cdot}74}{30}m_2 = 40{,}458 + \frac{\Delta b}{h}E$$

$$\text{FBC}\ldots 23{\cdot}60m_3 + 3{\cdot}06(m_2+m_3) + 16{\cdot}92m_3 + \frac{596{\cdot}74}{30}(m_1+m_2+m_3) = 40{,}458 + \frac{\Delta b}{h}E$$

$$\text{ABC}\ldots 16{\cdot}92m_1 - 23{\cdot}60m_2 + 23{\cdot}60m_3 + 16{\cdot}92m_3 + \frac{596{\cdot}74}{30}m_1 + \frac{596{\cdot}74}{30}(m_1+m_2+m_3) = 40{,}458 \times 2$$

$$\text{C}\ \ldots \frac{596{\cdot}74}{30}m_3 + \frac{596{\cdot}74}{30}m_3 + \frac{8375{\cdot}9}{30^2}(m_1+m_2+m_3) = \frac{1{,}240{,}668}{30} - \frac{2\Delta b}{h}E$$

from which $m_1 = 776{,}320$ ft.-lb., $m_2 = 326{,}570$ ft.-lb., and $m_3 = 375{,}670$ ft.-lb.

For a concentrated load, from (69):

$$\text{EAB}\ldots 23{\cdot}60m_1 + 3{\cdot}06m_1 + \frac{596{\cdot}74}{30}m_1 - 16{\cdot}92m_2 - \frac{596{\cdot}74}{30}m_2 + \frac{596{\cdot}74}{30}m_1 + \frac{8375{\cdot}9}{30^2}m_1 = \frac{\Delta b}{h}E$$

$$\text{ABC}\ldots 16{\cdot}92m_1 - 23{\cdot}60m_2 + 23{\cdot}60m_3 + 16{\cdot}92m_4 + \frac{596{\cdot}74}{30}m_1 + \frac{596{\cdot}74}{30}(m_1+m_2+m_3) = 3384{\cdot}5$$

$$\text{FBC}\ldots 23{\cdot}60m_3 + 3{\cdot}06(m_2+m_3) + 16{\cdot}92m_4 + \frac{596{\cdot}74}{30}(m_1+m_2+m_3) = 3384{\cdot}5 + \frac{\Delta b}{h}E$$

$$\text{C}\ \ldots \frac{596{\cdot}74}{30}m_3 + \frac{596{\cdot}74}{30}m_4 + \frac{8375{\cdot}9}{30^2}(m_1+m_2+m_3) = \frac{87{,}881{\cdot}9}{30} - \frac{\Delta b}{h}E - \frac{\Delta c}{h}E$$

$$\text{BCD}\ldots 16{\cdot}92m_3 + 23{\cdot}60m_4 - 23{\cdot}60m_5 + 16{\cdot}92m_6 + \frac{596{\cdot}74}{30}(m_1+m_2+m_3) + \frac{596{\cdot}74}{30}m_6 = 2461{\cdot}0$$

$$\text{BCG}\ldots 16{\cdot}92m_3 + 23{\cdot}60m_4 + 3{\cdot}06(m_4+m_5) + \frac{596{\cdot}74}{30}(m_1+m_2+m_3) = 2461{\cdot}0 + \frac{\Delta c}{h}E$$

$$\text{CDK}\ldots 23{\cdot}60m_6 + 3{\cdot}06m_6 - 16{\cdot}92m_5 - \frac{596{\cdot}74}{30}m_5 + \frac{596{\cdot}74}{30}m_6 + \frac{596{\cdot}74}{30}m_6 + \frac{8375{\cdot}9}{30^2}m_6 = \frac{\Delta c}{h}E$$

$$\text{S.c.}\ldots m_1 + m_2 + m_3 = m_4 + m_5 + m_6$$

from which $m_1 = 2560$ ft.-lb., $m_2 = 1160$ ft.-lb., $m_3 = 69{,}870$ ft.-lb., $m_4 = 9550$ ft.-lb., $m_5 = 38{,}140$ ft.-lb., $m_6 = 25{,}900$ ft.-lb.

Influence of Change of Temperature on Continuous Frames with Curved Members.

Equations (64), (67), and (70*a*), with slight modification, can be used in calculating thermal stresses in continuous frames with curved members.

Two-span Frames (*Fig.* 50).—For two-span frames the angular gaps and the deflected shape are shown in *Fig.* 50 by dotted lines, and the equations of equilibrium are:

$$\left.\begin{array}{l}
DAB \ldots m_1.\alpha_{ab} + m_1.\alpha_{ad} + m_1.\dfrac{\Delta_1^m}{h} - m_2.\beta_1 - m_2\dfrac{\Delta_1^m}{h} + H_d\,\tau_1 + H_d\,\dfrac{\Delta_1^h}{h} = \dfrac{\Delta t_1}{h} + \dfrac{\Delta b}{h} \\
ABC \ldots m_1.\beta_1 - m_2.\alpha_{ba} - m_3.\alpha_{bc} + m_4.\beta_2 + H_d.\delta_1 + H_f\,\tau_2 = 0 \\
ABE \ldots -m_1.\beta_1 + m_2.\alpha_{ba} + (m_3 - m_2)\alpha_{be} - H_d.\delta_1 = \dfrac{\Delta b}{h} \\
BCF \ldots m_4.\alpha_{cb} + m_4\,\alpha_{cf} - m_3.\beta_2 + m_4\dfrac{\Delta_2^m}{h} - m_3\dfrac{\Delta_2^m}{h} + H_f\,\delta_2 + H_f\,\dfrac{\Delta_2^h}{h} = \dfrac{\Delta t_2}{h} - \dfrac{\Delta b}{h} \\
S.c. \ldots m_1 + (m_3 - m_2) = m_4
\end{array}\right\} \quad . \quad (72)$$

in which $\Delta t = \alpha_t TL$, α_t is the coefficient of thermal expansion, T is the change of temperature, and other symbols are as defined previously.

Three-span Symmetrical Frames (*Fig.* 51).—Similarly, thermal stresses in three-span symmetrical frames are calculated from the following equations of equilibrium :

$$\left.\begin{array}{l}
EAB \ldots m_1.\alpha_{ab} + m_1.\alpha_{ae} - m_2.\beta_1 - m_2\dfrac{\Delta_1^m}{h} + m_1\dfrac{\Delta_1^m}{h} + H_e\,\tau_1 + H_e\dfrac{\Delta_1^h}{h} = \dfrac{\Delta t_1}{h} + \dfrac{\Delta b}{h} \\
ABC \ldots -m_1.\beta_1 + m_2\,\alpha_{ba} + m_3\,\alpha_{bc} + m_3.\beta_2 - H_e\,\delta_1 - (H_e + H_f).\tau_2 = 0 \\
ABF \ldots m_2.\alpha_{ba} + (m_2 - m_3)\alpha_{bf} - m_1.\beta_1 - H_e\,\delta_1 = \dfrac{\Delta b}{h} \\
B \quad \ldots -m_3.\Delta_2^m + (H_e + H_f)\dfrac{\Delta_2^h}{2} = \dfrac{\Delta t_2}{2} - \Delta b
\end{array}\right\} \quad . \quad (73)$$

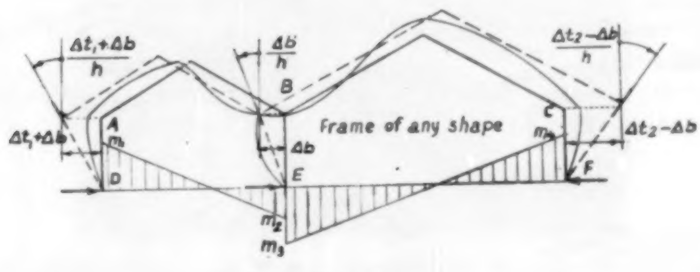


Fig. 50.

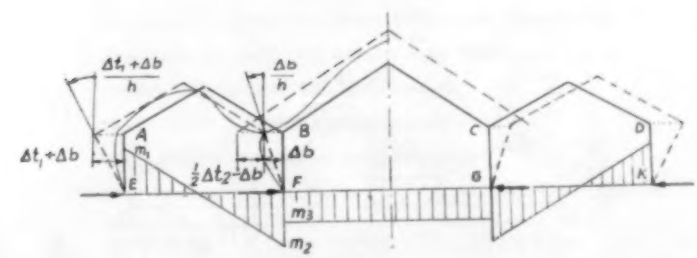


Fig. 51.

EXAMPLE.—As a numerical example, consider the frame shown in *Fig.* 52 subjected to change of temperature. Assuming that $T = 50$ deg. F., $\alpha_t = 0{\cdot}000006$, and $E = 4{,}000{,}000$ lb. per square inch (576×10^6 lb. per square foot), the values of Δt are calculated as follows:

$$EI\Delta t_1 = \pm\, 0{\cdot}000006 \times 50 \times 40 \times 576{,}000 \times 0{\cdot}666 = \pm\, 4607{\cdot}5.$$
$$EI\Delta t_2 = \pm\, 0{\cdot}000006 \times 50 \times 60 \times 576{,}000 \times 0{\cdot}666 = \pm\, 6911{\cdot}5.$$

The elastic constants are:

Span 1: $EI\alpha = 15{\cdot}55$; $EI\beta = 7{\cdot}775$; $EI\Delta^m = 139{\cdot}92$; $EI\Delta^h = 2240$.
Span 2: $EI\alpha = 21{\cdot}54$; $EI\beta = 10{\cdot}77$; $EI\Delta^m = 258{\cdot}5$; $EI\Delta^h = 5514{\cdot}53$.

Substituting these values in equations (72),

$$DAB \ldots 15{\cdot}55 m_1 + \frac{16}{3} m_1 - 7{\cdot}775 m_2 + \frac{139{\cdot}92}{16} m_1 - \frac{139{\cdot}92}{16} m_2 + \frac{139{\cdot}92}{16} m_1 + \frac{2240}{16^2} m_1 = \frac{4607{\cdot}5}{16} + \frac{\Delta b}{h} EI$$

$$ABC \ldots 7{\cdot}775 m_1 - 15{\cdot}55 m_2 - 21{\cdot}54 m_3 + 10{\cdot}77 m_4 + \frac{139{\cdot}92}{16} m_1 + \frac{258{\cdot}5}{16} m_4 = 0$$

$$ABE \ldots -7{\cdot}775 m_1 + 15{\cdot}55 m_2 + \frac{16}{3}(m_3 - m_2) - \frac{139{\cdot}92}{16} m_1 = \frac{\Delta b}{h} EI$$

$$BCF \ldots 21{\cdot}54 m_4 + \frac{16}{3} m_4 - 10{\cdot}77 m_3 + \frac{258{\cdot}5}{16} m_4 - \frac{258{\cdot}5}{16} m_3 + \frac{258{\cdot}5}{16} m_4 + \frac{5514{\cdot}53}{16^2} m_4 = \frac{6911{\cdot}5}{16} - \frac{\Delta b}{h} EI$$

$$S.c. \ldots m_1 - m_2 + m_3 - m_4 = 0$$

from which $m_1 = 10{,}409{\cdot}5$ ft.-lb., $m_2 = 11{,}944$ ft.-lb., $m_3 = 10{,}243{\cdot}5$ ft.-lb., and $m_4 = 8709$ ft.-lb.

The thermal stresses in the three-span frame shown in *Fig.* 53 are calculated from equations (73). The values of Δt and the elastic constants are:

Span 1:
$EI\Delta t_1 = 4607{\cdot}5$; $EI\alpha = 15{\cdot}55$; $EI\beta = 7{\cdot}775$; $EI\Delta^m = 139{\cdot}92$; $EI\Delta^h = 2240$.

Span 2:
$EI\alpha = 21{\cdot}54$; $EI\beta = 10{\cdot}77$; $EI\Delta^m = 258{\cdot}5$; $EI\Delta^h = 5514{\cdot}53$; $EI\Delta t_2 = 6911{\cdot}5$

and the equations of equilibrium become

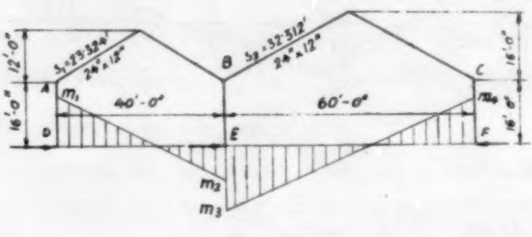


Fig. 52.

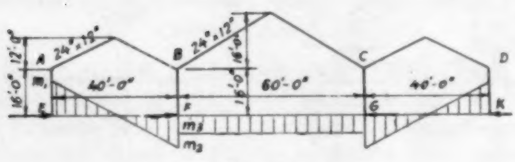


Fig. 53.

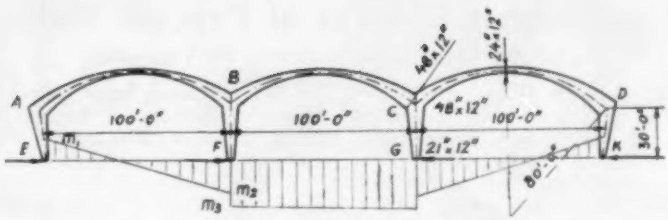


Fig. 54.

$$\text{EAB} \ldots 15{\cdot}55m_1 + \frac{16}{3}m_1 - 7{\cdot}775m_2 + \frac{139{\cdot}92}{16}m_1 - \frac{139{\cdot}92}{16}m_2 + \frac{139{\cdot}92}{16}m_1 + \frac{2240}{16^2}m_1 = \frac{4607{\cdot}5}{16} + \frac{\Delta b}{h}EI$$

$$\text{ABC} \ldots -7{\cdot}775m_1 + 15{\cdot}55m_2 + 21{\cdot}54m_3 + 10{\cdot}77m_3 - \frac{139{\cdot}92}{16}m_1 - \frac{258{\cdot}5}{16}(m_1 + m_2 - m_3) = 0$$

$$\text{ABF} \ldots 15{\cdot}55m_2 + \frac{16}{3}(m_2 - m_3) - 7{\cdot}775m_1 - \frac{139{\cdot}92}{16}m_1 = \frac{\Delta b}{h}EI$$

$$\text{B} \ldots -\frac{258{\cdot}5}{16}m_3 + \frac{5514{\cdot}53}{2 \times 16^2}(m_1 + m_2 - m_3) = \frac{6911{\cdot}5}{2 \times 16} - \frac{\Delta b}{h}EI$$

from which $m_1 = 16{,}100$ ft.-lb., $m_2 = 21{,}050$ ft.-lb., and $m_3 = 11{,}110$ ft.-lb.

Example.—The thermal stresses in the frame shown in *Fig.* 54 are calculated from equations (73). The elastic constants are :

Beam : $E\alpha = 23{\cdot}60$; $E\beta = 16{\cdot}92$; $E\Delta^m = E\gamma = 596{\cdot}74$; $E\Delta^h = 8375{\cdot}9$.
Column : $E\alpha = 3{\cdot}06$.

Assuming that $T = 50$ deg. F., $\alpha_t = 0{\cdot}000006$, and $E = 4{,}000{,}000$ lb. per square inch,

$$EI\Delta t = 0{\cdot}000006 \times 50 \times 100 \times 576{,}000 = 17{,}300,$$

and the equations of equilibrium become :

$$\text{EAB} \ldots 23{\cdot}60m_1 + 3{\cdot}06m_1 - 16{\cdot}92m_2 + \frac{596{\cdot}74}{30}m_1 - \frac{596{\cdot}74}{30}m_2 + \frac{596{\cdot}74}{30}m_1 + \frac{8375{\cdot}9}{30^2}m_1 = \frac{17{,}300}{30} + \frac{\Delta b}{h}E$$

$$\text{ABC} \ldots -16{\cdot}92m_1 + 23{\cdot}60m_2 + 23{\cdot}60m_3 + 16{\cdot}92m_3 - \frac{596{\cdot}74}{30}m_1 - \frac{596{\cdot}74}{30}(m_1 + m_3 - m_2) = 0$$

$$\text{ABF} \ldots 23{\cdot}60m_2 + 3{\cdot}06(m_3 - m_2) - 16{\cdot}92m_1 - \frac{596{\cdot}74}{30}m_1 = \frac{\Delta b}{h}E$$

$$\text{B} \ldots -\frac{596{\cdot}74}{30}m_3 + \frac{8375{\cdot}9}{2 \times 30^2}(m_1 + m_3 - m_2) = \frac{17{,}300}{2 \times 30} - \frac{\Delta b}{h}E$$

from which $m_1 = 39{,}091{\cdot}4$ ft.-lb., $m_2 = 68{,}660{\cdot}2$ ft.-lb., and $m_3 = 37{,}309{\cdot}3$ ft.-lb.

Lectures on Building.

The following lectures have been arranged by the Ministry of Works. Admission free.

Lightweight Concrete. By W. Kinniburgh. Crown and Anchor Hotel, Westgate Street, Ipswich. Nov. 15. 8 p.m.

Some Building Methods in the U.S.A. By W. R. Turner. Schofield Technical College, Park Road, Mexborough. Nov. 16. 7.15 p.m. Technical College, Hopwood Lane, Halifax. Nov. 23. 7.15 p.m.

Essentials of Good Concreting. By E. E. H. Bate. Technical College, Bolton. November 16. 7.15 p.m.

Prestressed Concrete. By F. Walley. Municipal College, Burnley. November 18. 7.15 p.m. By J. S. Arlett. Technical College, St. George Gate, Doncaster. November 30. 7.15 p.m.

Load-bearing Walls of Precast Slabs.

A LARGE HOUSING ESTATE IN FRANCE.

THE buildings shown in *Fig.* 1 comprise a housing estate, known as the "cité Rotterdam", in Strasbourg, France. The scheme was the subject of a competition organised towards the end of the year 1950 by the French Ministry of Reconstruction and Town Planning for the development of a site of about 25 acres to provide dwellings, with a total area exceeding half a million square feet, and schools. The sizes of the dwellings vary from about 250 sq. ft. to over 1000 sq. ft., more than half of them having areas between 600 and 750 sq. ft. The conditions imposed upon the competitors included a construction time of eighteen months and a limit of cost of about £1,300,000 at the prices current in France in January, 1951. Twenty-four designs were submitted and in July, 1951, the first prize was awarded to the architect M. Eugène Beaudouin who worked in collaboration with the contractors Entreprise Boussiron. This scheme provided 808 dwellings in eleven buildings, of which eight, not exceeding five stories high, have load-bearing walls, and three, up to thirteen stories high, have reinforced concrete frames.

Construction.

Where the sub-soil is gravel the structures with load-bearing walls have strip footings and the framed structures have reinforced concrete column bases joined by continuous beams. Over part of the site, however, the presence of old ditches required the use of piles. A cross section through a five-story structure with load-bearing walls on a piled foundation is shown in *Fig.* 2.

Above ground level the load-bearing walls are of cavity construction. They comprise an outer leaf, 7½ in. thick, of precast concrete slabs, an air space of about 1½ in., and an inner leaf, 4½ in. thick, of hollow clay blocks. The precast slabs of story-height consist of 6½ in. of "no-fines" concrete and a 1-in. face of normal concrete with an exposed aggregate finish. The inside face of the wall is plastered. *Fig.* 3 gives details of the load-bearing walls, and their external appearance is shown in *Fig.* 4. The floors are precast slabs containing hollow-tiles

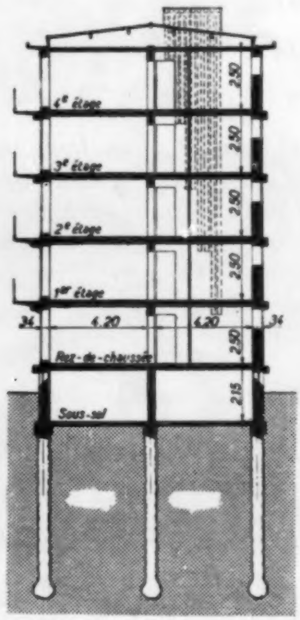
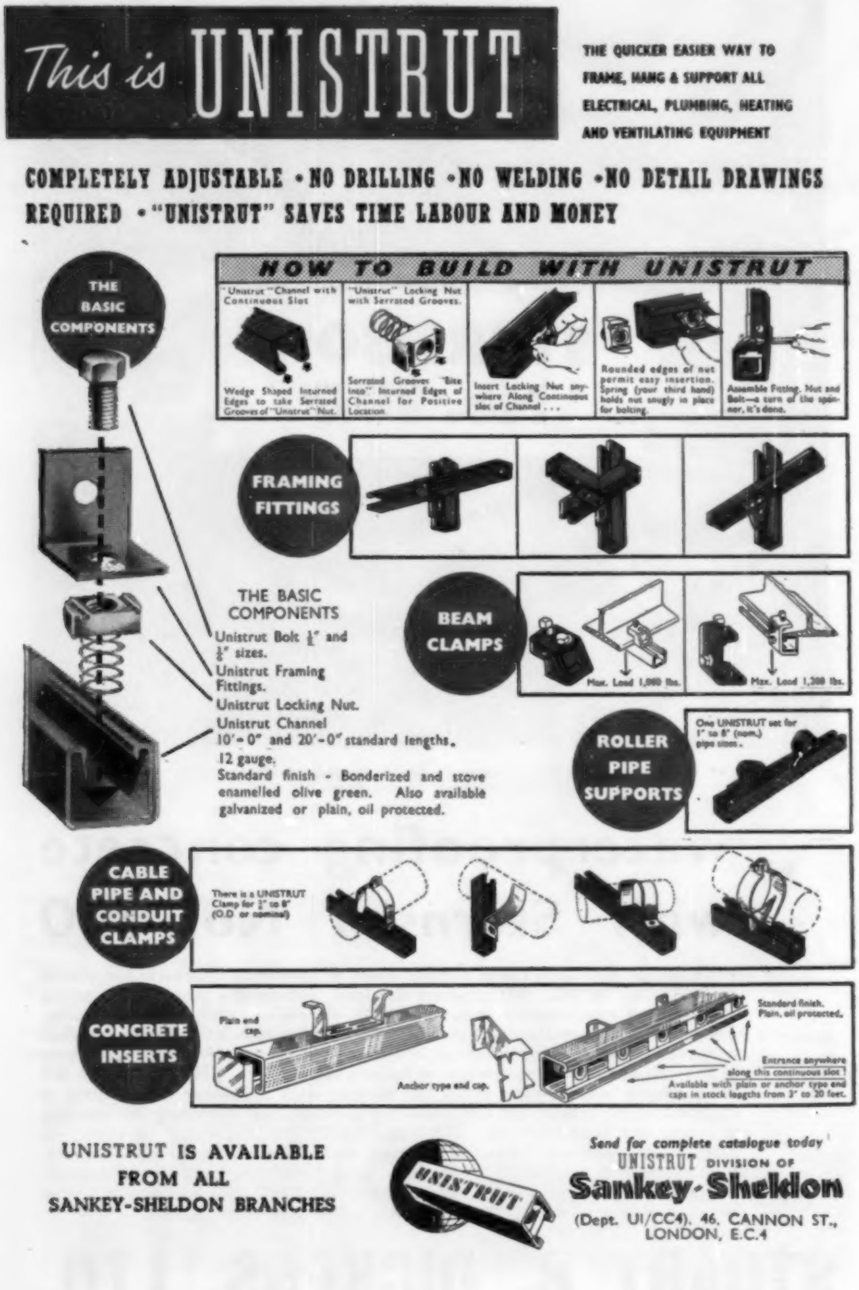


Fig. 2.—Section through 5-story Building with Load-bearing Walls.

Fig. 1.—A View of the Completed Estate.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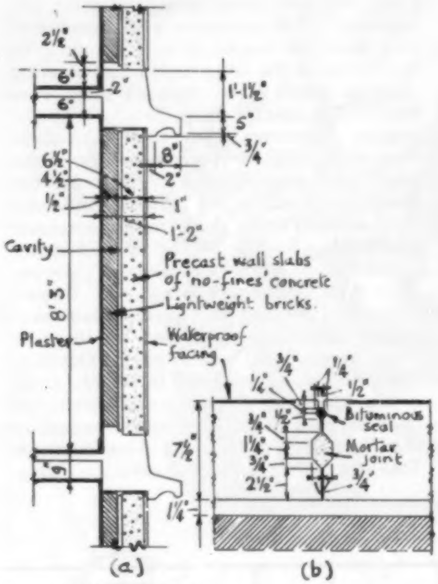

(a) Vertical Section.
(b) Horizontal Section showing joint between Precast Slabs.

Fig. 3.—Load-bearing Precast Wall.

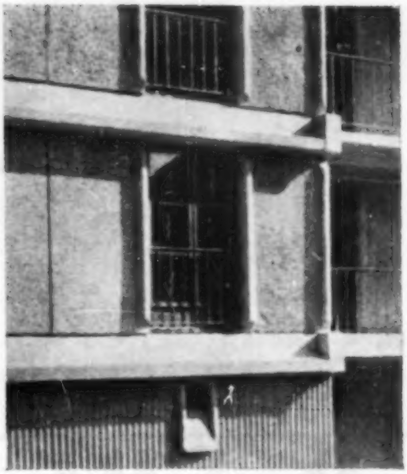

Fig. 4.—The External Face of a Load-bearing Wall of Precast Slabs.

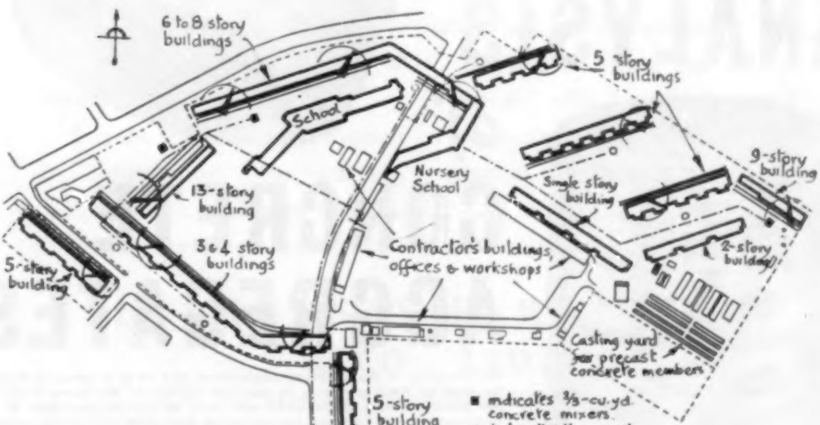

Fig. 5.—Arrangement of Plant.

with in-situ toppings in which the heating coils are placed. Precast concrete staircases of the type shown in *Fig.* 6 are used.

The arrangement of the site is shown in *Fig.* 5. The larger items of contractor's plant included eleven tower cranes and one crane mounted on a lorry, three concrete mixers each with a capacity of $\frac{3}{8}$ cu. yd., and seven mixers of $\frac{1}{2}$ cu. yd. capacity. The concrete was distributed by cranes on tracks of 2 ft. gauge. In the centre of the site was the bar-bending shop in which 1200 tons of steel were bent. The casting shop, in which all the precast members other than floor-slabs were made, was on the eastern side of the site. The floor-slabs were cast on the ground adjacent to the buildings in which they were to be used. The approximate quantities of some of the precast members required were: wall slabs, 23,900 sq. yd.; floor slabs, 59,800 sq. yd.; lintols, $5\frac{1}{3}$ miles. During the work a maximum of 900 men were employed on the site. Work commenced on the site in October, 1951, and was completed in March, 1953.

The foregoing description is abstracted from an article by M. André Bouchet in the Belgian journal "La Technique de Travaux" for January–February, 1954.

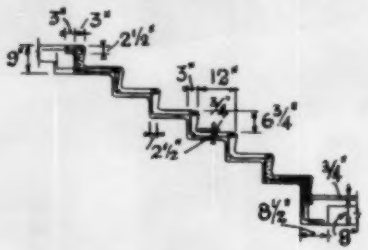

Fig. 6.—Section through Precast Stairs.

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The Bearing Capacities of Footings.

DURING a lecture by Mr. P. L. Capper on "Soil Mechanics in Relation to Structural Engineering," given at the Institution of Structural Engineers in February, 1953, the method due to Professor W. S. Housel of calculating the bearing pressure required to produce a given settlement of a footing was briefly mentioned, and in the subsequent discussion several speakers commented that the method was not sufficiently appreciated in this country.

The method is described in "Applied Soil Mechanics," by Professor Housel, from which the following is summarised. The method is based upon tests and relates the bearing pressure to the area and shape of a footing and the settlement. The tests showed that the total load (W) on the footing could be expressed as $W = mP + nA$, in which P is the perimeter of the footing, A is the area of the footing, n is the direct bearing pressure, and m is a force acting around the perimeter of the footing; m and n are determined empirically. By dividing the equation throughout by A the average unit pressure (w) on the area of the footing is obtained, that is $w = \frac{W}{A} = m\frac{P}{A} + n$. The area of the footing is thus eliminated and the ratio of the perimeter to the area $\left(\frac{P}{A}\right)$ is introduced. The effect of the forces around the perimeter is thus expressed as an equivalent uniformly-distributed pressure $\left(m\frac{P}{A}\right)$ which is added to the direct pressure (n). The tests showed that, for any given settlement, m and n are constant for each size of footing and depend on the ratio $\frac{P}{A}$. For example, for a footing 2 ft. square $\frac{P}{A} = 2$ and for a given settlement $w = 2m + n$, but for a footing 10 ft. square $\frac{P}{A} = 0{\cdot}4$ and for the same settlement $w = 0{\cdot}4m + n$. The difference between the bearing capacities is dependent upon the forces around the perimeter of the footing, and in sandy soils the difference would not be so great as in clays.

The constants m and n have to be determined and this may be done by load tests using at least two, and preferably more, plates of different sizes. For each size of plate the settlement is plotted against the load, and from the resulting curves the values of m and n for any particular settlement are calculated by substituting in the equation $w = m\frac{P}{A} + n$ the observed values of w and $\frac{P}{A}$, the equations thus obtained being solved simultaneously. Knowing m and n, the value of w for a given settlement of a proposed foundation can be calculated by trial and error.

Book Review.

"Erfahrungen mit Betonstrassen." By R. Dittrich. (Berlin: Wilhelm Ernst & Sohn. 1953. Price 12 D.M.)

THIS is a report of an investigation on the concrete carriageways of the German autobahnen constructed during the years 1935 to 1941. About one-third of the roads were inspected and records were made of all slabs having cracks easily visible. While some part of the roads were entirely free from cracks, in other parts up to 90 per cent. of all the slabs were cracked. The conclusions drawn appear to be as follows. The stability of the slabs is largely affected by the sub-grade. Large cracks caused by frost were unmistakable due to their diagonal and irregular course. Where transverse joints between slabs were not dowelled, wavy movements were observed at the ends of the slabs; this was reduced by increased consolidation of the subsoil, but some rocking was seen, particularly in short slabs. Reinforcement prevented cracking, entirely or at least to a great extent, even where the subsoil was poor and inferior cements had been used. The length of unreinforced bays is of particular importance where the subsoil is poor. Varying the lengths of bays, as was done during the first years of construction, in order to prevent the rocking effect produced by vehicles passing over the slabs, is not considered necessary. With a reasonably good foundation, adequate reinforcement, and dowelled transverse joints the length of slab recommended is 30 yd.

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(*Continued on page* lxvi).

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(*Continued from page* lxv.)

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